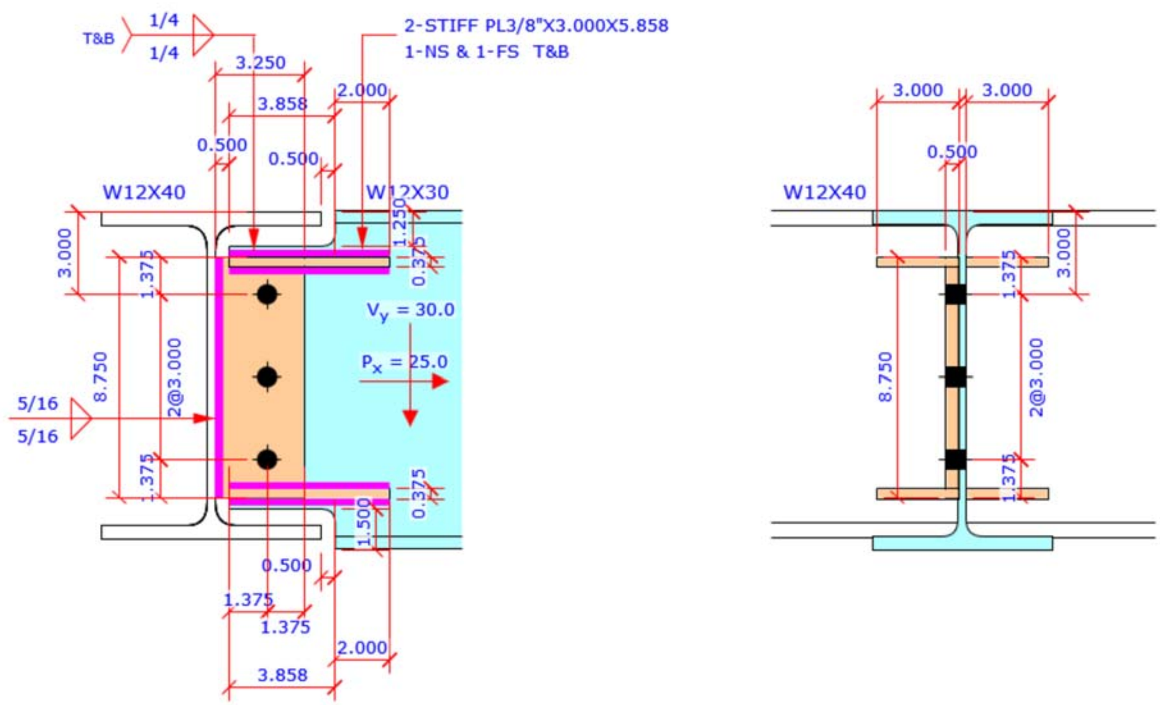
Beam to Girder	Shear Tab Shear Connection	Code=AISC 360-10 LRFD

Result Summary	geometries & weld limitations = **PASS**	limit states max ratio = **0.96**	**PASS**

Sketch	Shear Connection	Code=AISC 360-10 LRFD



Members & Components Summary		
Member	Shear Connection	Code=AISC 360-10 LRFD

Geometry Restriction Checks - Shear Tab to Beam Web					PASS
Min Bolt Edge Distance - Shear Tab to Beam Web					
Bolt diameter	d_b =		= 0.750	[in]	
Min edge distance allowed	$L_{e\text{-}min}$ =		= **1.000**	[in]	AISC 14th Table J3.4
Min edge distance in Shear Tab to Beam Web	L_e =		= **1.375**	[in]	
			> $L_{e\text{-}min}$	OK	
Min Bolt Spacing - Shear Tab to Beam Web					
Bolt diameter	d_b =		= 0.750	[in]	
Min bolt spacing allowed	$L_{s\text{-}min}$ = 2.667 d_b		= **2.000**	[in]	AISC 14th J3.3
Min Bolt spacing in Shear Tab to Beam Web	L_s =		= **3.000**	[in]	
			> $L_{s\text{-}min}$	OK	

Weld Limitation Check - Shear Tab Weld					PASS
Min Fillet Weld Size					
Thinner part joined thickness	t =		= 0.295	[in]	
Min fillet weld size allowed	w_{min} =		= **0.188**	[in]	AISC 14th Table J2.4
Fillet weld size provided	w =		= **0.313**	[in]	
			> w_{min}	OK	
Min Fillet Weld Length					
Fillet weld size provided	w =		= 0.313	[in]	
Min fillet weld length allowed	L_{min} = 4 x w		= **1.250**	[in]	AISC 14th J2.2b
Min fillet weld length	L =		= **8.750**	[in]	
			> L_{min}	OK	

W Shape Beam - Tensile Yield		ratio = 25.00 / 111.74	= **0.22**		PASS
Gross area subject to tension	A_g =		= 2.483	[in^2]	
Steel yield strength	F_y =		= 50.0	[ksi]	
Tensile force required	P_u =		= **25.00**	[kips]	
Tensile yielding strength	$R_n = F_y A_g$		= 124.15	[kips]	AISC 14th Eq D2-1
Resistance factor-LRFD	ϕ = 0.90				AISC 14th D2 (a)
	ϕR_n =		= **111.74**	[kips]	AISC 14th Eq D2-1
	ratio = **0.22**		> P_u	OK	

W Shape Beam - Tensile Rupture			ratio = 25.00 / 87.77	= **0.28**	**PASS**
W beam section	= W12X30				
	d = 12.300 [in]		b_f = 6.520	[in]	
	t_f = 0.440 [in]		t_w = 0.260	[in]	
	A_g = 2.483 [in^2]				
Bolt hole diameter	bolt dia d_b = $^3/_4$ [in]		bolt hole dia d_h = $^7/_8$	[in]	AISC 14th B4.3b
Number of bolt row	n = 3				
W section net area	$A_n = A_g - n\, d_h\, t_w$		= 1.801	[in^2]	
Shear lag factor	U =		= 1.000		AISC 14th D3
Tensile force required	P_u =		= **25.00**	[kips]	
Tensile effective net area	$A_e = A_n U$		= 1.801	[in^2]	
Plate tensile strength	F_u =		= 65.0	[ksi]	
Tensile rupture strength	$R_n = F_u A_e$		= 117.03	[kips]	AISC 14th Eq D2-2
Resistance factor-LRFD	ϕ = 0.75				AISC 14th D2 (b)
	ϕR_n =		= **87.77**	[kips]	AISC 14th Eq D2-2
	ratio = **0.28**		> P_u	OK	

Beam Web - Shear Yielding			ratio = 30.00 / 74.49	= **0.40**	**PASS**
Plate Shear Yielding Check					
Plate size	width b_p = 9.550 [in]		thickness t_p = 0.260	[in]	
Plate yield strength	F_y = 50.0 [ksi]				
Plate gross area in shear	$A_{gv} = b_p t_p$		= 2.483	[in^2]	
Shear force required	V_u =		= **30.00**	[kips]	
Plate shear yielding strength	$R_n = 0.6\, F_y A_{gv}$		= 74.49	[kips]	AISC 14th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00				AISC 14th Eq J4-3
	ϕR_n =		= **74.49**	[kips]	
	ratio = **0.40**		> V_u	OK	

Beam Web - Shear Rupture			ratio = 30.00 / 52.66	= **0.57**	**PASS**
Plate Shear Rupture Check					
Bolt hole diameter	bolt dia d_b = $^3/_4$ [in]		bolt hole dia d_h = $^7/_8$	[in]	AISC 14th B4.3b
Number of bolt	n = 3				
Plate size	width b_p = 9.550 [in]		thickness t_p = 0.260	[in]	
Plate tensile strength	F_u = 65.0 [ksi]				
Plate net area in shear	$A_{nv} = (\, b_p - n\, d_h\,)\, t_p$		= 1.801	[in^2]	
Shear force required	V_u =		= **30.00**	[kips]	
Plate shear rupture strength	$R_n = 0.6\, F_u A_{nv}$		= 70.22	[kips]	AISC 14th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75				AISC 14th Eq J4-4
	ϕR_n =		= **52.66**	[kips]	
	ratio = **0.57**		> V_u	OK	

Beam Web - Bolt Bearing on Beam Web	ratio = 39.05 / 53.68	= **0.73**	**PASS**

The bolt group is oriented so that the shear force V is in ver. direction and the axial force P is in hor. direction

Bolt group forces	shear V = 30.00 [kips]	axial P = -25.00	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **39.05**	[kips]	
Resultant force/hor line load angle	$\theta = \tan^{-1} (V / P)$	= 50.19	[°]	
Bolt hole diameter	bolt dia d_b = 0.750 [in]	bolt hole dia d_{bh} = 0.813	[in]	AISC 14th B4.3b
Bolt hole ver. dimension	d_v =	= 0.813	[in]	
Bolt hole hor. dimension	d_h =	= 0.813	[in]	
Bolt center to bolt hole edge dist	$d_c = 0.5\, d_{bh}$	= 0.406	[in]	
Bolt no in ver & hor direction	Bolt Row n_v = 3	Bolt Col n_h = 1		
Bolt spacing	ver s_v = 3.000 [in]			
Bolt edge distance	ver e_v = 1.750 [in]	hor e_h = 1.375	[in]	
Bolt clear dist - bot right corner bolt	$L_{cA} = \min \left(\dfrac{e_v}{\sin\theta} , \dfrac{e_h}{\cos\theta} \right) - d_c$	= 1.742	[in]	
Bolt clear dist - right side edge bolt	$L_{cB} = \min \left(\dfrac{s_v - 0.5 d_v}{\sin\theta} , \dfrac{e_h}{\cos\theta} \right) - d_c$	= 1.742	[in]	

Single Bolt Shear Strength

Bolt shear stress	bolt grade = A325-N	F_{nv} = 54.0	[ksi]	AISC 14th Table J3.2
	bolt dia d_b = 0.750 [in]	bolt area A_b = 0.442	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$	= 23.86	[kips]	AISC 14th Eq J3-1
Bolt bearing on plate	thick t = 0.260 [in]	tensile F_u = 65.0	[ksi]	
Bolt bearing strength	$R_{n\text{-}br} = 3.0\, d_b\, t\, F_u$	= 38.03	[kips]	AISC 14th Eq J3-6b
Type A - Bolt Group Bottom Right Corner Bolt				
Number of bolt	n_A = 1			
Bolt tear out strength	$R_{n\text{-}tA} = 1.5\, L_{cA}\, t\, F_u$	= 44.15	[kips]	AISC 14th Eq J3-6b
Bolt bearing strength	$R_{nA} = \min (R_{n\text{-}tA} , R_{n\text{-}br} , R_{n\text{-}bolt})$	= **23.86**	[kips]	
Type B - Bolt Group Right Side Edge Bolt				
Number of bolt	n_B = 2			
Bolt tear out strength	$R_{n\text{-}tB} = 1.5\, L_{cB}\, t\, F_u$	= 44.15	[kips]	AISC 14th Eq J3-6b
Bolt bearing strength	$R_{nB} = \min (R_{n\text{-}tB} , R_{n\text{-}br} , R_{n\text{-}bolt})$	= **23.86**	[kips]	
Bolt bearing strength for all bolts	$R_n = n_A R_{nA} + n_B R_{nB} + n_C R_{nC} + n_D R_{nD}$	= 71.57	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 14th J3-10
	ϕR_n =	= **53.68**	[kips]	
	ratio = **0.73**	> R	OK	

Beam Web - Shear - Block Shear - 1-Side Strip		ratio = 30.00 / 54.19	= **0.55**	**PASS**

Plate Block Shear - Side Strip

Bolt hole diameter	bolt dia $d_b = {}^3/_4$	[in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 14th B4.3b
Plate thickness	$t_p = 0.260$	[in]			
Plate strength	$F_y = 50.0$	[ksi]	$F_u = 65.0$	[ksi]	
Bolt no in ver & hor dir	$n_v = 1$		$n_h = 3$		
Bolt spacing in hor dir	$s_h = 3.000$	[in]			
Bolt edge dist in ver & hor dir	$e_v = 1.375$	[in]	$e_h = 1.750$	[in]	
Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p$		= 2.015	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [\ (n_h - 1) + 0.5\ ]\ d_h t_p$		= 1.446	[in^2]	
Net area subject to tension	$A_{nt} = (\ e_v - 0.5\ d_h\ )\ t_p$		= 0.244	[in^2]	
Block shear strength required	$V_u =$		= **30.00**	[kips]	
Uniform tension stress factor	$U_{bs} = 1.00$				AISC 14th Fig C-J4.2
Bolt shear resistance provided	$R_n = \min(0.6F_u A_{nv}, 0.6F_y A_{gv}) + U_{bs} F_u A_{nt}$		= 72.25	[kips]	AISC 14th Eq J4-5
Resistance factor-LRFD	$\phi = 0.75$				AISC 14th Eq J4-5
	$\phi R_n =$		= **54.19**	[kips]	
	ratio = **0.55**		> V_u	OK	

Beam Web - Axial Tearout - Block Shear - Center Strip		ratio = 25.00 / 68.13	= **0.37**	**PASS**

Plate Block Shear - Center Strip

Bolt hole diameter	bolt dia $d_b = {}^3/_4$	[in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 14th B4.3b
Plate thickness	$t_p = 0.260$	[in]			
Plate strength	$F_y = 50.0$	[ksi]	$F_u = 65.0$	[ksi]	
Bolt no in ver & hor dir	$n_v = 3$		$n_h = 1$		
Bolt spacing in ver & hor dir	$s_v = 3.000$	[in]	$s_h = 3.000$	[in]	
Bolt edge dist in ver & hor dir	$e_v = 3.000$	[in]	$e_h = 1.375$	[in]	
Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p \times 2$		= 0.715	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5]\ d_h t_p \times 2$		= 0.488	[in^2]	
Net area subject to tension when sheared out by center strip	$A_{nt} = (\ n_v - 1)\ (\ s_v - d_h\ )\ t_p$		= 1.105	[in^2]	
Block shear strength required	$V_u =$		= **25.00**	[kips]	
Uniform tension stress factor	$U_{bs} = 1.00$				AISC 14th Fig C-J4.2
Bolt shear resistance provided	$R_n = \min(0.6F_u A_{nv}, 0.6F_y A_{gv}) + U_{bs} F_u A_{nt}$		= 90.84	[kips]	AISC 14th Eq J4-5
Resistance factor-LRFD	$\phi = 0.75$				AISC 14th Eq J4-5
	$\phi R_n =$		= **68.13**	[kips]	
	ratio = **0.37**		> V_u	OK	

Coped Beam - Flexural Rupture			ratio = 30.00 / 240.69	= **0.12**	**PASS**
Beam section & cope side	sect = W12X30		cope side = double cope		
Beam top flange cope	depth d_c = 1.250	[in]	length L_c = 3.858	[in]	
Beam bottom flange cope	depth d_c = 1.500	[in]	length L_c = 3.858	[in]	
S_{net} of Coped Beam With Hor Reinforcing Stiffener Plates					
Beam sect W12X30	d = 12.300	[in]	b_f = 6.520	[in]	
	t_f = 0.440	[in]	t_w = 0.260	[in]	
Stiffener plate size	w_p = 3.000	[in]	t_p = 0.375	[in]	
Flange cope depth-top & bot flange	d_{ct} = 1.250	[in]	d_{cb} = 1.500	[in]	
Properties of Coped W Sect With Hor Reinforcing Stiffener Plates					
Top flange	b_{ft} = 6.260	[in]	t_{ft} = 0.375	[in]	
Bottom flange	b_{fb} = 6.260	[in]	t_{fb} = 0.375	[in]	
W sect depth	d = 8.800	[in]	web t_w = 0.260	[in]	
Dist from sect centroid to T&B flange face	x_t = 4.400	[in]	x_b = 4.400	[in]	
Max dist sect centroid to T&B flange face	$x_{max} = \max(x_t, x_b)$		= 4.400	[in]	
W sect moment of inertia	I_x =		= 94.7	[in^4]	
W sect elastic modulus	$S_{net} = I_x / x_{max}$		= **21.52**	[in^3]	
Beam section tensile strength	F_u =		= 65.0	[ksi]	
Distance from face of cope to the point of inflection of beam	e =		= 4.358	[in]	AISC 14th Page 9-6
Beam end shear force	V_u =		= **30.00**	[kips]	
Beam end shear resistance	$R_n = F_u S_{net} / e$		= 320.92	[kips]	AISC 14th Eq 9-4
Resistance factor-LRFD	ϕ = 0.75				AISC 14th Eq 9-4
	ϕR_n =		= **240.69**	[kips]	
	ratio = **0.12**		> V_u	OK	

Coped Beam - Local Web Buckling			ratio = 30.00 / 222.17	= **0.14**	**PASS**
Beam section & cope side	sect = W12X30		cope side = double cope		
Beam top flange cope	depth d_{ct} = 1.250	[in]	length L_{ct} = 3.858	[in]	
Beam bottom flange cope	depth d_{cb} = 1.500	[in]	length L_{cb} = 3.858	[in]	
S_{net} of Coped Beam With Hor Reinforcing Stiffener Plates					
Beam sect W12X30	d = 12.300	[in]	b_f = 6.520	[in]	
	t_f = 0.440	[in]	t_w = 0.260	[in]	
Stiffener plate size	w_p = 3.000	[in]	t_p = 0.375	[in]	
Flange cope depth-top & bot flange	d_{ct} = 1.250	[in]	d_{cb} = 1.500	[in]	
Properties of Coped W Sect With Hor Reinforcing Stiffener Plates					
Top flange	b_{ft} = 6.260	[in]	t_{ft} = 0.375	[in]	
Bottom flange	b_{fb} = 6.260	[in]	t_{fb} = 0.375	[in]	
W sect depth	d = 8.800	[in]	web t_w = 0.260	[in]	
Dist from sect centroid to T&B flange face	x_t = 4.400	[in]	x_b = 4.400	[in]	
Max dist sect centroid to T&B flange face	$x_{max} = \max(x_t, x_b)$		= 4.400	[in]	
W sect moment of inertia	I_x =		= 94.7	[in^4]	
W sect elastic modulus	$S_{net} = I_x / x_{max}$		= **21.52**	[in^3]	
Distance from face of cope to the point of inflection of beam	e =		= 4.358	[in]	AISC 14th Page 9-6
Beam section	depth d = 12.300	[in]	web t_w = 0.260	[in]	
	F_y = 50.0	[ksi]	E = 29000	[ksi]	
	$f_d = 3.5 - 7.5\,(d_{ct} / d)$		= 2.738		AISC 14th Eq 9-13
Reduced beam depth	$h_0 = d - d_{ct} - d_{cb}$		= 9.550	[in]	
Plate local buckling stress	$F_{cr} = 0.62\,\pi\,E\,\dfrac{t_w^2}{L_{ct}\,h_0}\,f_d$		= 283.7	[ksi]	AISC 14th Eq 9-12
	$F_{cr} = F_{cr} \le F_y$		= 50.0	[ksi]	AISC 14th Eq 9-12
Beam end shear force	V_u =		= **30.00**	[kips]	
Beam end shear resistance	$R_n = F_{cr}\,S_{net} / e$		= 246.86	[kips]	AISC 14th Eq 9-6
Resistance factor-LRFD	ϕ = 0.90				AISC 14th Eq 9-6
	$\phi\,R_n$ =		= **222.17**	[kips]	
	ratio = **0.14**		> V_u	OK	

Hor Stiffener to Coped Beam Web Fillet Weld Limitation					**PASS**
Min Fillet Weld Size					
Thinner part joined thickness	$t =$		= 0.260	[in]	
Min fillet weld size allowed	$w_{min} =$		= **0.188**	[in]	AISC 14th Table J2.4
Fillet weld size provided	$w =$		= **0.250**	[in]	
			> w_{min}	OK	
Min Fillet Weld Length					
Fillet weld size provided	$w =$		= 0.250	[in]	
Min fillet weld length allowed	$L_{min} = 4 \times w$		= **1.000**	[in]	AISC 14th J2.2b
Min fillet weld length	$L =$		= **5.858**	[in]	
			> L_{min}	OK	
Hor Reinforcing Stiffener Extension Beyond Cope					
To prevent local crippling of the beam web, the longitudinal stiffening must be extended min a distance of d_c beyond the cope					AISC 14th Fig 9-10 (b)
Flange cope depth-top & bot flange	$d_{ct} = 1.250$ [in]		$d_{cb} = 1.500$	[in]	
Max cope depth - top & bot flange	$d_c = \max(d_{ct}, d_{cb})$		= **1.500**	[in]	
Hor stiffener plate extension beyond cope	$L_e =$		= **2.000**	[in]	
			> d_c	OK	

Hor Stiffener to Coped Beam Web Fillet Weld Strength		ratio = 3.13 / 10.97	= **0.29**	**PASS**

Stiffener to Coped Beam Web Weld Line Force Calc

Refer to AISC Design Example v15 Page IIA-78 for the formula used below on how to get the stiffener weld line forces

From S_{net} calc in Coped Beam - Local Web Buckling check above, the properties of stiffener reinforced W section

Reinforced W sect moment of inertia	I_{net} =	= 94.7	[in^4]	
Reinforced stiffener plate area	A_p =	= 2.348	[in^2]	
Dist from centroid of reinforced sect to centroid of stiffener plate	y =	= 4.213	[in]	
First moment of reinforced stiffener plate	$Q = A_p\, y$	= 9.892	[in^3]	
Beam end shear force	V_u =	= **30.00**	[kips]	
Weld line shear stress	$r_{u1} = \dfrac{V_u\, Q}{I_{net}}$	= 3.134	[kip/in]	
Distance from face of cope to the point of inflection of beam	e =	= 4.358	[in]	AISC 14th Page 9-6
Beam web hor coped length	L_c =	= 3.858	[in]	
Hor stiffener plate extension beyond the cope	L_e =	= 2.000	[in]	
Stiffener to beam web weld length	$L_w = L_c + L_e$	= 5.858	[in]	
Weld line shear stress	$r_{u2} = \dfrac{V_u\, e\, Q}{I_{net}\, L_w}$	= 2.331	[kip/in]	
Weld line shear stress - max	$r_u = \max(r_{u1}, r_{u2})$	= **3.134**	[kip/in]	

Fillet Weld Strength Calc

Fillet weld leg size	$w = ^1/_4$ [in]	load angle θ = 0.0	[°]	
Electrode strength	F_{EXX} = 70.0 [ksi]	strength coeff C_1 = 1.00		AISC 14th Table 8-3
Number of weld line	n = 2 for double fillet			
Load angle coefficient	$C_2 = (1 + 0.5 \sin^{1.5}\theta)$	= 1.00		AISC 14th Page 8-9
Fillet weld shear strength	$R_{n\text{-}w} = 0.6\,(C_1 \times 70\text{ ksi})\,0.707\,w\,n\,C_2$	= 14.847	[kip/in]	AISC 14th Eq 8-1
Base metal - stiffener	thickness t = 0.375 [in]	tensile F_u = 65.0	[ksi]	
Base metal - stiffener is in shear, shear rupture as per AISC 14th Eq J4-4 is checked				AISC 14th J2.4
Base metal shear rupture	$R_{n\text{-}b} = 0.6\, F_u\, t$	= 14.625	[kip/in]	AISC 14th Eq J4-4
Double fillet linear shear strength	$R_n = \min(R_{n\text{-}w}, R_{n\text{-}b})$	= **14.625**	[kip/in]	AISC 14th Eq 9-2
Resistance factor-LRFD	φ = 0.75			AISC 14th Eq 8-1
	ϕR_n =	= **10.969**	[kip/in]	
	ratio = **0.29**	> r_u	OK	

Shear Tab - Shear Yielding			ratio = 30.00 / 131.25	= **0.23**	**PASS**
Plate Shear Yielding Check					
Plate size	width b_p = 8.750	[in]	thickness t_p = 0.500	[in]	
Plate yield strength	F_y = 50.0	[ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$		= 4.375	[in^2]	
Shear force required	V_u =		= **30.00**	[kips]	
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$		= 131.25	[kips]	AISC 14th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00				AISC 14th Eq J4-3
	ϕR_n =		= **131.25**	[kips]	
	ratio = **0.23**		> V_u	OK	

Shear Tab - Shear Rupture			ratio = 30.00 / 89.58	= **0.33**	**PASS**
Plate Shear Rupture Check					
Bolt hole diameter	bolt dia d_b = 3/4	[in]	bolt hole dia d_h = 7/8	[in]	AISC 14th B4.3b
Number of bolt	n = 3				
Plate size	width b_p = 8.750	[in]	thickness t_p = 0.500	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate net area in shear	$A_{nv} = (b_p - n\, d_h)\, t_p$		= 3.063	[in^2]	
Shear force required	V_u =		= **30.00**	[kips]	
Plate shear rupture strength	$R_n = 0.6 F_u A_{nv}$		= 119.44	[kips]	AISC 14th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75				AISC 14th Eq J4-4
	ϕR_n =		= **89.58**	[kips]	
	ratio = **0.33**		> V_u	OK	

Shear Tab - Axial Tensile Yield			ratio = 25.00 / 196.88	= **0.13**	**PASS**
Plate Tensile Yielding Check					
Plate size	width b_p = 8.750	[in]	thickness t_p = 0.500	[in]	
Plate yield strength	F_y = 50.0	[ksi]			
Plate gross area in shear	$A_g = b_p t_p$		= 4.375	[in^2]	
Tensile force required	P_u =		= **25.00**	[kips]	
Plate tensile yielding strength	$R_n = F_y A_g$		= 218.75	[kips]	AISC 14th Eq J4-1
Resistance factor-LRFD	ϕ = 0.90				AISC 14th Eq J4-1
	ϕR_n =		= **196.88**	[kips]	
	ratio = **0.13**		> P_u	OK	

Shear Tab - Axial Tensile Rupture			ratio = 25.00 / 149.30	= **0.17**	**PASS**	
Plate Tensile Rupture Check						
Bolt hole diameter	bolt dia $d_b = 3/4$	[in]	bolt hole dia $d_h = 7/8$	[in]		AISC 14th B4.3b
Number of bolt	n = 3					
Plate size	width b_p = 8.750	[in]	thickness t_p = 0.500	[in]		
Plate tensile strength	F_u = 65.0	[ksi]				
Plate net area in tension	$A_{nt} = (b_p - n\ d_h)\ t_p$		= 3.063	[in^2]		
Tensile force required	P_u =		= **25.00**	[kips]		
Plate tensile rupture strength	$R_n = F_u A_{nt}$		= 199.06	[kips]		AISC 14th Eq J4-2
Resistance factor-LRFD	ϕ = 0.75					AISC 14th Eq J4-2
	ϕR_n =		= **149.30**	[kips]		AISC 14th Eq J4-2
	ratio = **0.17**		> P_u	OK		

Shear Tab - Flexural Yield Interact			ratio =	= **0.12**	**PASS**	
Plate width & thick	width b_p = 8.750	[in]	thick t_p = 0.500	[in]		
	yield F_y = 50.0	[ksi]				
Shear plate - gross area	$A_g = b_p \times t_p$		= 4.375	[in^2]		
Shear plate - plastic modulus	$Z_p = (b_p \times t_p^2) / 4$		= 9.570	[in^3]		
Axial strength available	P_c = from axial tensile yield check		= 196.88	[kips]		
Axial strength required	P_r = from user load input		= 25.00	[kips]		
Shear strength available	V_c = from shear yielding check		= 131.25	[kips]		
Shear strength required	V_r = from user load input		= 30.00	[kips]		
Flexural strength available	$M_c = \phi F_y Z_p$ ϕ=0.90		= 35.89	[kip-ft]		
Bolt group ecc for shear V_r	e_v =		= 1.875	[in]		
Bolt group ecc for axial P_r	e_p =		= 0.000	[in]		
Flexural strength required	$M_r = V_r e_v + P_r e_p$		= 4.69	[kip-ft]		
Flexural yield interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$		= **0.12**			AISC 14th Eq 10-5
			< 1.0	OK		

Shear Tab - Flexural Rupture Interact		ratio =	= **0.22**	**PASS**	
Plate A_n and Z_{net} Calc					
Bolt hole diameter	bolt dia $d_b = {}^3/_4$ [in]	bolt hole dia $d_h = {}^7/_8$	[in]		AISC 14th B4.3b
Number of bolt	$n = 3$				
Plate size	width $b_p = 8.750$ [in]	thickness t_p = 0.500	[in]		
Plate net area	$A_n = (b_p - n\ d_h)\ t_p$	= 3.063	[in^2]		
Plate net plastic sect modulus	$Z_{net} =$	= 6.850	[in^3]		
Plate net elastic sect modulus	$S_{net} =$	= 4.561	[in^3]		
Plate width & thick	width $b_p = 8.750$ [in]	thick t_p = 0.500	[in]		
	tensile $F_u = 65.0$ [ksi]				
Axial strength available	P_c = from axial tensile rupture check	= 149.30	[kips]		
Axial strength required	P_r = from user load input	= 25.00	[kips]		
Shear strength available	V_c = from shear rupture check	= 89.58	[kips]		
Shear strength required	V_r = from user load input	= 30.00	[kips]		
Flexural strength available	$M_c = \phi\ F_u\ Z_{net}$ ϕ=0.75	= 27.83	[kip-ft]		AISC 14th Eq 9-4
Bolt group ecc for shear V_r	$e_v =$	= 1.875	[in]		
Bolt group ecc for axial P_r	$e_p =$	= 0.000	[in]		
Flexural strength required	$M_r = V_r\ e_v + P_r\ e_p$	= 4.69	[kip-ft]		
Flexural rupture interaction	$\text{ratio} = \left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$	= **0.22**			AISC 14th Eq 10-5
		< 1.0	OK		

Shear Tab - Bolt Bearing on Shear Tab		ratio = 39.05 / 53.68	= **0.73**	**PASS**	
The bolt group is oriented so that the shear force V is in ver. direction and the axial force P is in hor. direction					
Bolt group forces	shear V = 30.00 [kips]	axial P = -25.00	[kips]		
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **39.05**	[kips]		
Resultant force/hor line load angle	$\theta = \tan^{-1} (V / P)$	= 50.19	[°]		
Bolt hole diameter	bolt dia d_b = 0.750 [in]	bolt hole dia d_{bh} = 0.813	[in]	AISC 14th B4.3b	
Bolt hole ver. dimension	d_v =	= 0.813	[in]		
Bolt hole hor. dimension	d_h =	= 0.813	[in]		
Bolt center to bolt hole edge dist	$d_c = 0.5\, d_{bh}$	= 0.406	[in]		
Bolt no in ver & hor direction	Bolt Row n_v = 3	Bolt Col n_h = 1			
Bolt spacing	ver s_v = 3.000 [in]				
Bolt edge distance	ver e_v = 1.375 [in]	hor e_h = 1.375	[in]		
Bolt clear dist - bot right corner bolt	$L_{cA} = \min \left(\frac{e_v}{\sin\theta}, \frac{e_h}{\cos\theta} \right) - d_c$	= 1.384	[in]		
Bolt clear dist - right side edge bolt	$L_{cB} = \min \left(\frac{s_v - 0.5 d_v}{\sin\theta}, \frac{e_h}{\cos\theta} \right) - d_c$	= 1.742	[in]		
Single Bolt Shear Strength					
Bolt shear stress	bolt grade = A325-N	F_{nv} = 54.0	[ksi]	AISC 14th Table J3.2	
	bolt dia d_b = 0.750 [in]	bolt area A_b = 0.442	[in²]		
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$	= 23.86	[kips]	AISC 14th Eq J3-1	
Bolt bearing on plate	thick t = 0.500 [in]	tensile F_u = 65.0	[ksi]		
Bolt bearing strength	$R_{n\text{-}br} = 3.0\, d_b\, t\, F_u$	= 73.13	[kips]	AISC 14th Eq J3-6b	
Type A - Bolt Group Bottom Right Corner Bolt					
Number of bolt	n_A = 1				
Bolt tear out strength	$R_{n\text{-}tA} = 1.5\, L_{cA}\, t\, F_u$	= 67.45	[kips]	AISC 14th Eq J3-6b	
Bolt bearing strength	$R_{nA} = \min (R_{n\text{-}tA}, R_{n\text{-}br}, R_{n\text{-}bolt})$	= **23.86**	[kips]		
Type B - Bolt Group Right Side Edge Bolt					
Number of bolt	n_B = 2				
Bolt tear out strength	$R_{n\text{-}tB} = 1.5\, L_{cB}\, t\, F_u$	= 84.90	[kips]	AISC 14th Eq J3-6b	
Bolt bearing strength	$R_{nB} = \min (R_{n\text{-}tB}, R_{n\text{-}br}, R_{n\text{-}bolt})$	= **23.86**	[kips]		
Bolt bearing strength for all bolts	$R_n = n_A R_{nA} + n_B R_{nB} + n_C R_{nC} + n_D R_{nD}$	= 71.57	[kips]		
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 14th J3-10	
	ϕR_n =	= **53.68**	[kips]		
	ratio = **0.73**	> R	OK		

Shear Tab - Beam Side - Block Shear - 1-Side Strip		ratio = 30.00 / 98.72	= **0.30**	**PASS**	

Plate Block Shear - Side Strip

Bolt hole diameter	bolt dia $d_b = {}^3/_4$ [in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 14th B4.3b	
Plate thickness	$t_p = 0.500$ [in]				
Plate strength	$F_y = 50.0$ [ksi]	$F_u = 65.0$	[ksi]		
Bolt no in ver & hor dir	$n_v = 1$	$n_h = 3$			
Bolt spacing in hor dir	$s_h = 3.000$ [in]				
Bolt edge dist in ver & hor dir	$e_v = 1.375$ [in]	$e_h = 1.375$	[in]		
Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p$	= 3.688	[in^2]		
Net area subject to shear	$A_{nv} = A_{gv} - [\ (n_h - 1) + 0.5\ ]\ d_h\ t_p$	= 2.594	[in^2]		
Net area subject to tension	$A_{nt} = (\ e_v - 0.5\ d_h\ )\ t_p$	= 0.469	[in^2]		
Block shear strength required	$V_u =$	= **30.00**	[kips]		
Uniform tension stress factor	$U_{bs} = 1.00$			AISC 14th Fig C-J4.2	
Bolt shear resistance provided	$R_n = \min\ (0.6F_u A_{nv}\ ,\ 0.6F_y A_{gv}\ ) + U_{bs} F_u A_{nt}$	= 131.63	[kips]	AISC 14th Eq J4-5	
Resistance factor-LRFD	$\phi = 0.75$			AISC 14th Eq J4-5	
	$\phi\ R_n =$	= **98.72**	[kips]		
	ratio = **0.30**	$> V_u$	OK		

Shear Tab - Beam Side-Axial Tearout - Block Shear - Center Strip		ratio = 25.00 / 131.02	= **0.19**	**PASS**	

Plate Block Shear - Center Strip

Bolt hole diameter	bolt dia $d_b = {}^3/_4$ [in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 14th B4.3b	
Plate thickness	$t_p = 0.500$ [in]				
Plate strength	$F_y = 50.0$ [ksi]	$F_u = 65.0$	[ksi]		
Bolt no in ver & hor dir	$n_v = 3$	$n_h = 1$			
Bolt spacing in ver & hor dir	$s_v = 3.000$ [in]	$s_h = 3.000$	[in]		
Bolt edge dist in ver & hor dir	$e_v = 1.375$ [in]	$e_h = 1.375$	[in]		
Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p \times 2$	= 1.375	[in^2]		
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5]\ d_h\ t_p \times 2$	= 0.938	[in^2]		
Net area subject to tension when sheared out by center strip	$A_{nt} = (\ n_v - 1)\ (\ s_v - d_h\ )\ t_p$	= 2.125	[in^2]		
Block shear strength required	$V_u =$	= **25.00**	[kips]		
Uniform tension stress factor	$U_{bs} = 1.00$			AISC 14th Fig C-J4.2	
Bolt shear resistance provided	$R_n = \min\ (0.6F_u A_{nv}\ ,\ 0.6F_y A_{gv}\ ) + U_{bs} F_u A_{nt}$	= 174.69	[kips]	AISC 14th Eq J4-5	
Resistance factor-LRFD	$\phi = 0.75$			AISC 14th Eq J4-5	
	$\phi\ R_n =$	= **131.02**	[kips]		
	ratio = **0.19**	$> V_u$	OK		

Shear Tab - Block Shear - Shear / Tensile Interact	ratio =		= **0.13**	**PASS**
Shear block shear strength required	V_u =	= 30.00	[kips]	
Axial block shear strength required	P_u =	= 25.00	[kips]	
Shear block shear strength available	ϕR_{nv} = from calc shown above	= 98.72	[kips]	
Axial block shear strength available	ϕR_{nt} = from calc shown above	= 131.02	[kips]	
Block shear shear/tensile interaction	ratio = $\left(\frac{V_u}{\phi R_{nv}}\right)^2 + \left(\frac{P_u}{\phi R_{nt}}\right)^2$	= **0.13**		AISC 14^{th} Eq 10-5
		< 1.0	OK	

Shear Tab - Lateral Stability / Stabilizer Plate	ratio = 39.05 / 1319.47		= **0.03**	**PASS**
Applied shear/axial forces	shear V = 30.00 [kips]	axial P = -25.00	[kips]	
Resultant shear force	$V_u = (V^2 + P^2)^{0.5}$	= **39.05**	[kips]	
Distance from support to the first line of bolts	a =	= 1.875	[in]	
Plate thickness & depth	t_p = 0.500 [in]	L = 8.750	[in]	
Shear resistance provided	$R_n = 1500\ \pi \frac{L\ t_p^3}{a^2}$	= 1466.08	[kips]	AISC 14^{th} Eq 10-6
Resistance factor-LRFD	ϕ = 0.90			AISC 14^{th} Eq 10-6
	ϕR_n =	= **1319.47**	[kips]	
	ratio = **0.03**	> V_u	OK	

Shear Tab - Plate Flexural Buckling	ratio = 30.00 / 109.47		= **0.27**	**PASS**
Shear tab size	depth = 8.750 [in]	thick = 0.500	[in]	
Plate buckling model	c = dist from support to fisrt bolt line	= 1.875	[in]	AISC 14^{th} Fig. 9-3
	h_0 = shear tab depth	= 8.750	[in]	
	t_w = shear tab thick	= 0.500	[in]	
Shear tab steel yield stress	F_y = 50.0 [ksi]			
Plate buckling factor	$\lambda = \frac{h_0 \sqrt{F_y}}{10\ t_w \sqrt{475 + 280\ (h_0 / c)^2}}$	= 0.153		AISC 14^{th} Eq 9-18
Plate buckling factor	Q =	= 1.000		AISC 14^{th} Eq 9-15
Plate critical buckling stress	$F_{cr} = Q\ F_y$	= 50.0	[ksi]	AISC 14^{th} Eq 9-14
Shear force in demand	V_u =	= **30.00**	[kips]	
Shear tab net elastic modulus	S_{net} =	= 4.561	[in^3]	
Shear force to bolt group CG ecc	a =	= 1.875	[in]	
Shear resistance	$R_n = F_{cr}\ S_{net} / a$	= 121.63	[kips]	AISC 14^{th} Eq 9-19
Resistance factor-LRFD	ϕ = 0.90			AISC 14^{th} Eq 9-19
	ϕR_n =	= **109.47**	[kips]	
	ratio = **0.27**	> V_u	OK	

Bolt Group Eccentricity					
Bolt group forces	shear V = 30.00	[kips]	axial P = 25.00	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$		= 39.05	[kips]	
Resultant force to ver Y axis angle	$\theta = \tan^{-1} (P / V)$		= 39.81	[°]	
Bolt group row and column	bolt row n_r = 3		bolt col n_c = 1		
Bolt row spacing	bolt row s_r = 3.000	[in]			
Shear force to bolt group CG ecc	e_x =		= 1.875	[in]	
Shear force to ver Y axis angle	θ =		= 39.81	[°]	
Bolt group coefficient C	C = from AISC 14th Table 7-6 ~ 7-13		= 2.267		
Bolt group eccentricity coefficient	$C_{ec} = C / (n_r \times n_c)$		= **0.756**		

Shear Tab / Beam Web - Bolt Shear			ratio = 39.05 / 40.58	= **0.96**	**PASS**
Bolt group forces	shear V = 30.00	[kips]	axial P = -25.00	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$		= **39.05**	[kips]	
Bolt shear stress	bolt grade = A325-N		F_{nv} = 54.0	[ksi]	AISC 14th Table J3.2
	bolt dia d_b = 0.750	[in]	bolt area A_b = 0.442	[in^2]	
Number of bolt carried shear	n_s = 3.0		shear plane m = 1		
Bolt group eccentricity coefficient	C_{ec} = from 'Bolt Group Eccentricity' calc		= 0.756		
Required shear strength	V_u =		= **39.05**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$		= 54.11	[kips]	AISC 14th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 14th Eq J3-1
	ϕR_n =		= **40.58**	[kips]	
	ratio = **0.96**		$> V_u$	OK	

Shear Tab to Girder Web Weld Strength		ratio = 8.03 / 14.63	= **0.55**	**PASS**	
Weld Group Forces					
	Shear V = 30.00 [kips]	Axial P = -25.00	[kips]	in tension	
Shear force to bolt group CG ecc	e_x =	= 1.875	[in]		
Moment due to eccentric shear V	$M = V \times e_x$	= 4.69	[kip-ft]		
Shear tab weld length	L =	= 8.750	[in]		
Combined Weld Stress					
Weld stress from axial force	$f_a = P / L$	= -2.857	[kip/in]	in tension	
Weld stress from shear force	$f_v = V / L$	= 3.429	[kip/in]		
Weld stress from moment force	$f_b = \dfrac{M}{L^2 / 6}$	= 4.408	[kip/in]		
Weld stress combined - max	$f_{max} = [\ (f_a - f_b)^2 + f_v^2\ ]^{0.5}$	= **8.034**	[kip/in]	AISC 14th Eq 8-11	
Weld stress load angle	$\theta = \tan^{-1} \left(\dfrac{f_a - f_b}{f_v} \right)$	= 64.7	[°]		
Fillet Weld Strength Calc					
Fillet weld leg size	$w = {}^{5}/_{16}$ [in]	load angle θ = 64.7	[°]		
Electrode strength	F_{EXX} = 70.0 [ksi]	strength coeff C_1 = 1.00		AISC 14th Table 8-3	
Number of weld line	n = 2 for double fillet				
Load angle coefficient	$C_2 = (\ 1 + 0.5 \sin^{1.5} \theta\ )$	= 1.43		AISC 14th Page 8-9	
Fillet weld shear strength	$R_{n\text{-}w} = 0.6\ (C_1 \times 70\ \text{ksi})\ 0.707\ w\ n\ C_2$	= 26.539	[kip/in]	AISC 14th Eq 8-1	
Base metal - shear tab	thickness t = 0.500 [in]	tensile F_u = 65.0	[ksi]		
Base metal - shear tab is in shear, shear rupture as per AISC 14th Eq J4-4 is checked				AISC 14th J2.4	
Base metal shear rupture	$R_{n\text{-}b} = 0.6\ F_u\ t$	= 19.500	[kip/in]	AISC 14th Eq J4-4	
Double fillet linear shear strength	$R_n = \min(\ R_{n\text{-}w}, R_{n\text{-}b}\ )$	= **19.500**	[kip/in]	AISC 14th Eq 9-2	
Resistance factor-LRFD	ϕ = 0.75			AISC 14th Eq 8-1	
	$\phi\ R_n$ =	= **14.625**	[kip/in]		
	ratio = **0.55**	> f_{max}	OK		